Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 50 No. 1 March 2019 ISSN 0046-5828

Validation of Foundation Design Method on Expansive Soils

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ABSTRACT: Nelson et al. (2015) presented design principles for foundations on expansive soils. The design principles consider free-field heave throughout the design life of a structure as the basis for foundation design. The design principles also consider water migration in the vadose zone, and the time required for subsoil wetting over the design life of the structure. This paper presents a method to validate th foundation design method presented in Nelson et al. (2015). The validation was performed using detailed long-term data obtained on a building constructed on expansive soils at the Denver International Airport, Denver, Colorado, USA. Water migration in the vadose zone and heave of floor slabs and drilled pier foundations were monitored over the time period from 2000 to 2016 and extended to a 25 year period (1991 - 2016) beginning at the end of construction. Water content profiles were modeled using VADOSE/W software, and heave of slabs and piers were computed using the design method presented in Nelson et al. (2015). The depth of wetting and changes in water content were used to compute heave according to the design method. Calculated heave was compared to the survey data. It was shown that the design method was capable of predicting heave to within 30 percent of the measured heave over a 25-year period.

KEYWORDS: Expansive Soils, Heave Prediction, Foundation Design, Water Migration Modeling

1. INTRODUCTION

The design of foundations for structures sited on expansive soils is one of the most challenging aspects of foundation engineering. Developers and owners of structures demand that the foundations perform within tolerable movement limits, yet at the same time, they are reluctant to spend the additional funds required to reduce the risk of intolerable movement. For expansive soil sites, the site investigation must be more extensive, the foundation design is more complex, and the foundation will cost more than that for ordinary soil sites.

On an expansive soil site, the foundation is often designed to account for the maximum heave that can occur. Although this is appropriate for sites where the depth of potential heave is not large, there are some cases where heave can extend to large depths, and the time required for the maximum heave to occur may exceed the design life of the structure. In those cases, it is appropriate to base the design on the amount of heave that will occur within the design life. Thus, the design of foundations for expansive soils must consider the time-wise migration of soil water in the vadose zone, and the extent to which subsoil wetting will occur during the design life of the structure.

Nelson et al. (2015) presented foundation design principles for structures on expansive soils. The design method considers the migration of subsurface water in the vadose zone, and the associated heave that occurs over the design life of the structure. Their method is briefly presented in the following sections of this paper. The design method was validated in this paper over a period of sixteen years at the Terminal Radar Approach Control (TRACON) site in Denver, Colorado, USA. It was then applied to predict the performance of the foundation over the design life of the structure.

The TRACON building that was analyzed is located in an area along the eastern edge of the Rocky Mountains in Colorado, USA. This area is commonly termed the “Front Range.” In this area, the expansive materials mainly comprise highly overconsolidated clays deposited during the Cretaceous period and soils derived from the weathering of those materials. The highly overconsolidated materials are referred to as claystone or clayshale bedrock. In this paper, the “bedrock” and soils will all be referred to as expansive soil.

In contrast to other areas such as the southwestern United States, Texas, Australia, and others where the soils exhibit swell when wetted and significant shrinkage when dried, the Front Range expansive soils tend to continue to heave over time with the negligible amount of shrinkage. Therefore, this paper relates only to the situation where the soils do not dry or shrink. Also, it does not address soils susceptible to hydrocollapse.

2. FOUNDATION DESIGN METHOD BY NELSON ET AL. (2015)

The Nelson et al. (2015) foundation design method is briefly presented in this section to provide an overview of the method. The detailed description of the method can be found in Nelson et al. (2015). The Nelson et al. (2015) method considers the migration of wetting front in expansive soils, and relates soil and foundation heave to water content changes. The importance of considering the time-wise migration of soil water for the design of foundations on highly expansive soils is illustrated in this paper by introducing a water migration model with either good or poor site drainage conditions. The results of the subsoil water migration and progression of heave with time for the model are presented in the following sections.

2.1 Free-Field Heave Prediction Using Oedometer Methods

Free-field heave distribution with depth is the primary data on which pier heave is calculated. Therefore, a review of free-field heave prediction methods is presented below.

Various heave prediction methods have been developed based on results of one-dimensional oedometer tests (Fredlund et al., 1980; U.S. Army Corps of Engineers, 1983; Nelson and Miller, 1992; Fredlund and Rahardjo, 1993; Fredlund et al., 2012). These methods utilize the net mechanical stress, $\sigma'' = (\sigma - u_a)$, and the matric suction; $h = (u_a - u_w)$ as the stress state variables. In these variables, σ is the total stress and u_a and u_w are the pore air and pore water pressures. The soil heave takes place as the suction is decreased. These methods are commonly referred to as “oedometer” methods.

2.1.1 Oedometer Methods

Two basic types of oedometer tests including the consolidation-swell (CS) test and the constant volume (CV) test are commonly performed for expansive soils. For the CS test, the sample is initially subjected to a prescribed vertical stress in the oedometer, and inundated under that constant vertical stress, σ''_i. The vertical strain that occurs due to wetting, termed the percent swell, $\varepsilon_{s\%}$, is measured, and the sample is usually subjected to an additional vertical load. The stress that would be required to restore the sample to its original height is termed the CS swelling pressure, σ''_{cs}. For the CV test, the sample is initially subjected to a prescribed vertical stress, but during inundation the sample is confined from swelling and the stress that is required to prevent the sample from swelling is measured. This is termed the CV swelling pressure, σ''_{cv}. The results of the CS and CV tests are normally plotted in the form of vertical strain as a function of the applied stress. The typical two-dimensional forms of each test are shown in Figure 1.

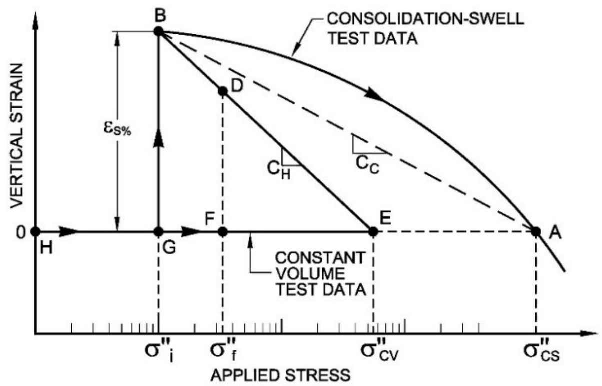


Figure 1 Oedometer test results and determination of heave index, C_H (Nelson et al., 2015)

2.1.2 Free-Field Heave Prediction

The oedometer methods have all used the same basic equation for calculation of heave. The general equation for predicting free-field heave is shown in Equation (1).

$$\rho_o = \sum_{i=1}^{n} \Delta H_i = \sum_{i=1}^{n} \left\{ C_H H log \left[\frac{\sigma''_{cv}}{\sigma''_f} \right] \right\}_i \tag{1}$$

where: ρ_o = total free-field heave,
ΔH_i = heave of layer i,
C_H = heave index of layer i,
H_i = initial thickness of layer i ,
σ''_{cv} = constant volume swelling pressure of layer i, and
σ''_f = final vertical net normal stress at the midpoint of layer i.

The parameter C_H defines the amount by which a soil sample will swell when it becomes wetted. This method considers both the change in suction due to wetting and the applied stress that is acting on the soil when it is wetted. The parameter C_H is the slope of the line BDE in Figure 1 and is equal to:

$$C_H = \frac{\varepsilon_{s\%}}{100 \times \log \left[\dfrac{\sigma''_{cv}}{\sigma''_i} \right]} \tag{2}$$

where: $\varepsilon_{s\%}$ = percent swell corresponding to the particular value of σ''_i expressed as a percent, and
σ''_i = vertical stress at which the sample is inundated.

2.1.3 Relationships Between σ''_{cv} and σ''_{cs}

The value of C_H can be determined from the results of a consolidation-swell test and a constant-volume test using identical samples of the same soil. However, in practice it is virtually impossible to obtain two identical samples from the field. Therefore, it is convenient to utilize a relationship between σ''_{cs} and σ''_{cv} such that C_H can be determined from a single consolidation-swell test.

A number of investigators including Edil & Alanazy (1992), Reichler (1997), Bonner (1998), Thompson et al. (2006), Nelson et al. (2006, 2012a) have proposed relationships between σ''_{cs} and σ''_{cv}. Those investigators generally propose some form of equation that utilizes a simple ratio between σ''_{cs} and σ''_{cv}. Nelson and Chao (2014) presented another approach by observing the nature of oedometer test results from a series of tests performed on identical samples and inundated under different values of inundation stress, σ''_i. They proposed a relationship between σ''_{cs} and σ''_{cv} as follows.

$$\log \sigma''_{cv} = \frac{\log \sigma''_{cs} + m \times \log \sigma''_i}{1+m} \tag{3}$$

where: m = slope of σ''_{cs} and σ''_i relationship

The parameter m depends on the particular soil, its expansive nature, and other properties of the soil. Nelson and Chao (2014) compiled a database of corresponding values of σ''_{cs} and σ''_{cv} based on various sources including their own laboratory data, review of many soils reports, and values published in the literature (Gilchrist, 1963; Porter, 1977; Reichler, 1997; Feng et al., 1998; Bonner, 1998; Fredlund, 2004; Al-Mhaidib, 2006; Thompson et al., 2006). Nelson and Chao (2014) concluded that most of the values of m varied between 0.4 and 1.2 with an average value of approximately 0.6.

2.1.4 Percent Swell and Swelling Pressure for Partially Wetted Soils

For the CS test, the soil sample is initially inundated under a constant vertical stress, σ''_i. Therefore, the values of percent swell and swelling pressure measured in the test represent a fully wetted soil condition. For partially wetted soils, it is necessary to modify the oedometer data for the fully wetted sample to determine the values of percent swell and swelling pressure to be used to predict heave. Figure 2 presents normalized percent swell plotted against degree of saturation for various values of the initial degree of saturation (modified from Chao, 2007). The normalized percent swell was determined by dividing the values of percent swell that occurred at a particular degree of saturation by the maximum values of percent swell obtained for fully wetted samples in the CS test. These curves can be used to calculate the percent swell for a partially wetted soil using the results of a fully wetted soil in the CS test. For example, a soil sample had an initial degree of saturation of 66% and swelled by 5.0% for the consolidation-swell test. The 5% swell represents the maximum percent swell value for a fully wetted soil condition (near a 100% degree of saturation). If the degree of saturation of the sample goes up to only 90%, the reduced percent swell for that sample can be calculated to be 4.2% (= 5% × 0.84) using the initial degree of saturation curve of 66% shown in Figure 2.

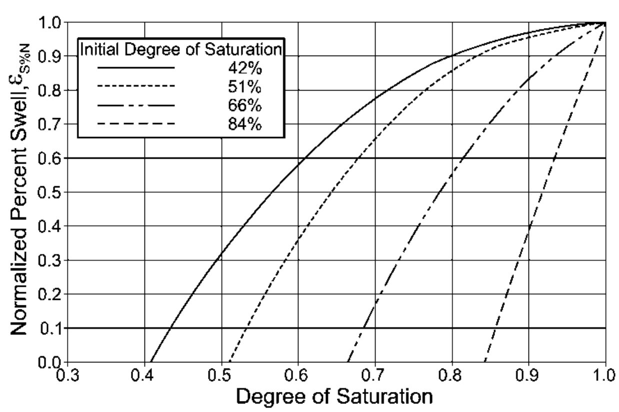


Figure 2 Normalized percent swell versus degree of saturation (modified from Chao, 2007)

In addition to determining a value of percent swell for a partial wetted soil, the swelling pressure for the soil must also be determined. Reichler (1997) indicated that an e-log p curve from the CS test for a partially wetted soil has the same shape as that for a fully wetted soil. This suggests a procedure for determining a value of reduced swelling pressure, σ''_{cvN}, to be used for computing heave in a partially wetted soil, as shown in Figure 3. To determine the reduced swelling pressure a line is drawn parallel to the C_H line and passing through the value of normalized percent swell, $\varepsilon_{s\%N}$, that was determined by Figure 2. Where that line intersects the axis for zero swell is the reduced swelling pressure, σ''_{cvN}.

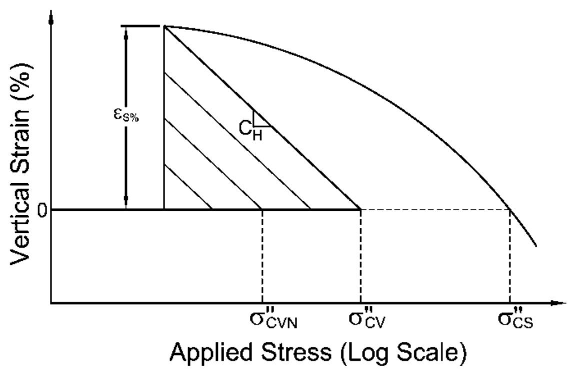


Figure 3 Procedure for determining swelling pressure for partially wetted soil (Nelson et al., 2015)

2.2 Deep Foundation Heave Prediction

The rigid pier method is commonly used by geotechnical practicing engineers to design piers in expansive soils. This method determines a required pier length by equating the downward skin friction below the depth of the design active zone, plus the dead load, to the uplift pressures exerted on the pier by the expansive soil. Chen (1988), O'Neill (1988), and Nelson and Miller (1992) present a detailed description of this method. The rigid pier design generally produces conservative pier lengths for a light structure founded on highly expansive soils. The rigid pier design works well if the stratum of the expansive soils is not thick and is underlain by a stable non-expansive stratum. However, in a deep deposit of expansive soils, the required pier length approaches a value equal to about twice the depth of the design active zone. In such cases the design rigid pier length is generally not practical for a light structure.

In reality structures are able to tolerate some amounts of pier movement. The amount of tolerable heave to be used for design depends on the nature of the structure. Methods of analysis of pier heave were developed by Poulos and Davis (1980) and were adapted by Nelson and Miller (1992) to develop design charts for calculating pier heave. This method is termed the elastic pier method. The elastic pier method was developed for piers with uniform properties with depth installed in a uniform soil profile. Additionally, the elastic pier analysis formulation breaks down when the length to diameter ratio becomes too great.

Finite element approaches to pier analyses provide versatility to consider such details as non-uniform soil or pier interface properties with depth and large length to diameter ratios. Finite element numerical analysis of pier heave in expansive soils has been proposed previously by several authors in the literature (Amir and Sokolov, 1976; Lytton 1977; Justo et al. 1984; Abdel-Halim and Al-Qasem, 1995; Mohamedzein et al. 1999). Nelson et al. (2012b) presents one such finite element based numerical analysis approach termed APEX for Analysis of Piers in EXpansive Soils.

Nelson et al. (2015) utilized the APEX program to develop design charts for use in facilitating pier design for sites where the soil conditions can be represented by a simplified heave profile. The design charts are presented in Figure 4. The charts were prepared for two types of cumulative free-field heave distributions. The shapes of the two types of distributions are shown in the insets on Figures 4a and 4b. The top segment of the two distributions shown in the insets is a vertical line of depth D_o. This represents the depth of a layer of non-expansive fill. If such a layer does not exist, the value of D_o is zero. The Type A profile was prepared for the case where the expansion potential of the soil below the depth D_o is constant with depth. In that case the cumulative heave profile has a logarithmic shape, as shown in Figure 4a. The Type B profile shown in Figure 4b was prepared for the case where non-expansive soil exists at the top of the soil layer below which the expansion potential below the depth D_o increases with depth.

Figure 4 presents the ratio of pier heave to free-field heave, ρ_p/ρ_o, as a function of the term $(L\text{-}D_o)/z_{AD}$, where L is the pier length and z_{AD} is the design active zone. The design active zone is defined as the zone of soil that is expected to become wetted by the end of design life (Nelson et al., 2015). The design curves were developed for a zero dead load condition. In Figure 4, a unitless parameter E_A was introduced to express the modulus of elasticity, E_s, of the soil. The modulus of elasticity, E_s, of the soil is expressed in terms of atmospheres. Thus, $E_A = E_s$ / 1 atmosphere, where atmospheric pressure is expressed in the same unit as E_s.

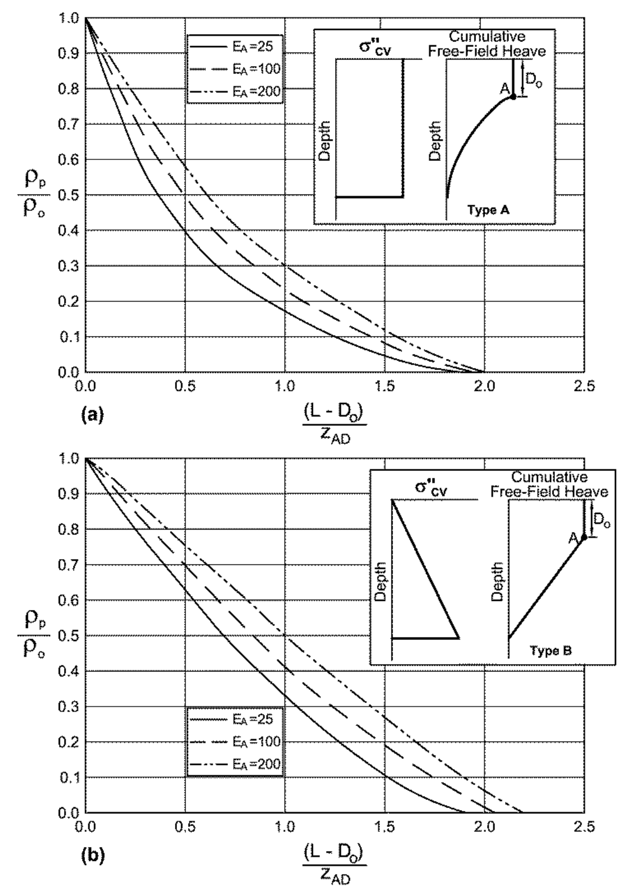


Figure 4 Design charts for piers in expansive soils: (a) σ''_{cv} constant with depth; (b) σ''_{cv} increasing with depth (Nelson et al., 2015)

2.3 Subsoil Water Migration and Progression of Heave with Time

The progression of free-field heave and foundation heave with time is an important consideration for establishing a value of the predicted heave to be used for final foundation design. Prediction of the time-wise progression of heave considers the migration of water in the zone of potential heave, and relates free-field heave to water content changes. For soil profiles in which the depth of wetting that will occur during the design life of the structure is less than the depth of potential heave, the design value of foundation heave may be less than the maximum potential heave.

Figure 5 shows a "wetting front" moving downward with time due to infiltration. For expansive soils with relatively low values of hydraulic conductivity, the wetting front typically transitions over some distance from a higher water content above the wetting front to that of the native soil below as shown in Figure 5. Because the transition zone may contribute significantly to heave, the depth of wetting, z_w, is defined as the depth to the bottom of the transition zone.

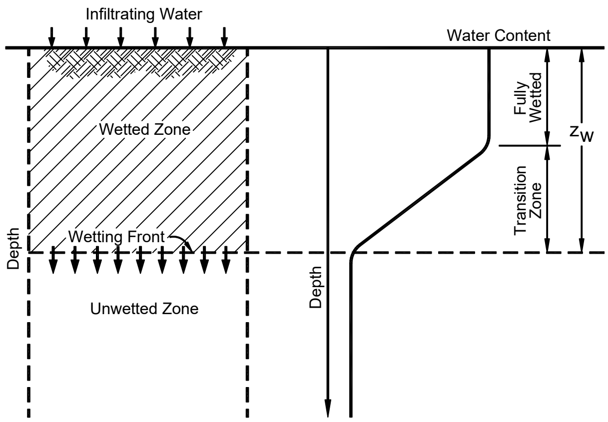


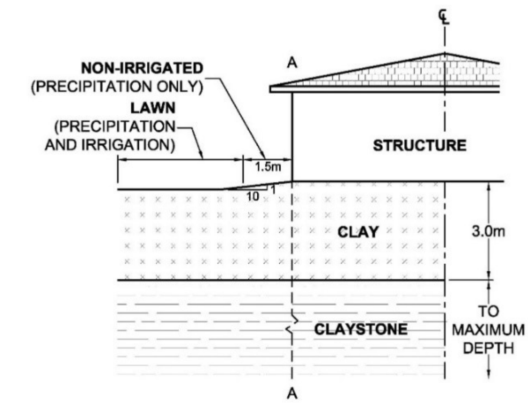
Figure 5 Progression of Wetting in Unsaturated Soil (Modified from McWhorter and Nelson, 1979)

Commercially available software (e.g., VADOSE/W; SVFLUX; MODFLOW-SURFACT) is available that can take into account complex climatological conditions, heterogeneous soil conditions, and wetting from both the ground surface and deep water sources. The computed subsoil wetting profiles can be used, along with considerations of reduced heave for soils that are not fully wetted using the approach described in Section 2.1.4, to compute heave as a function of time.

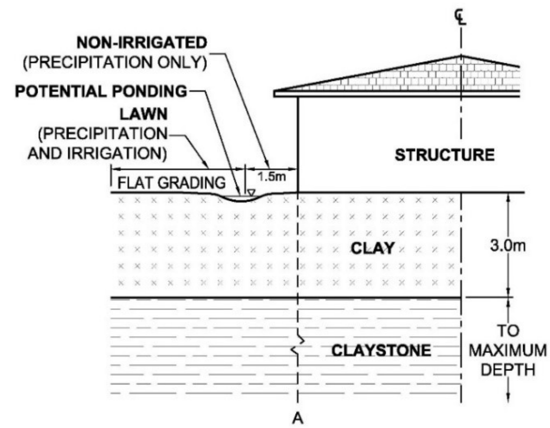
Factors that influence the migration of the wetting zone include, among others, climatological factors, irrigation, and surface grading and drainage. To demonstrate the importance of surface grading and drainage, analyses of subsoil water migration were performed for different surface drainage conditions as shown in Figure 6. These analyses were conducted by using the computer program VADOSE/W (GEO-SLOPE, 2007). In Figure 6a, the slope of the site grading is 10% for the first 3.0 meters away from the building. The surface is covered with non-irrigated gravel for a distance of 1.5 meters from the building. In Figure 6b, the grading is flat and the ponding is assumed to be full during the periods of irrigation and empty at other times. A major factor that influences infiltration is micro-ponding. This is caused by irregularities in the ground surface that produce small depressions which contribute to micro-ponding (Nelson et al., 2011).

The computed water content profiles along Section A defined in Figure 6 are presented in Figure 7. Comparison of the water content profiles shown in Figure 7 shows the importance of maintaining good surface grading. The results indicate that for ideal drainage conditions partial wetting will continue to migrate downward to a depth of approximately 12 meters in 100 years. For poor drainage conditions, water will migrate to that depth in 40 years, and in 100 years water can migrate to a depth of about 20 meters. This shows the dramatic effect that surface grading can have on surface infiltration of water.

The depth of potential heave for the site depicted in Figure 6 was computed to be 16.3 meters. From Figure 7, it is seen that for good drainage conditions the foundation soil would not be fully wetted by the end of the design life of the structure at 100 years. Thus, for a site such as this, which has relatively low permeable claystone and well maintained surface conditions, the foundation could be designed for a value of "design heave" less than the maximum heave.



(a) Ideal Surface Drainage Conditions



(b) Poor Surface Drainage Conditions

Figure 6 Cross Sections Used for Water Migration Modeling

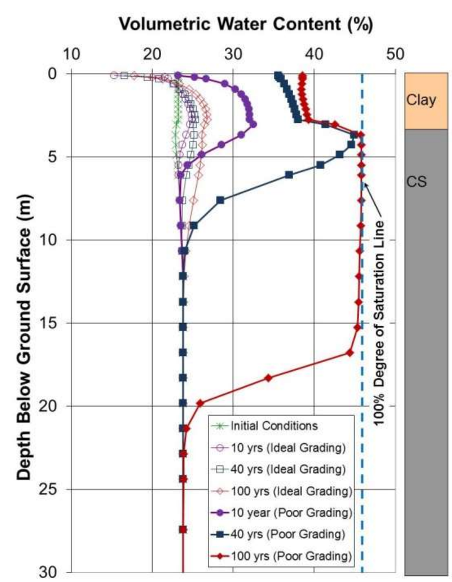


Figure 7 Comparison of Long-Term Water Content Profiles Along Profile A for Ideal and Poor Drainage Conditions

Using Equation (1) with the final water content data shown in Figure 7, the soil expansion properties shown in Table 1, and taking into consideration reduced heave for non-fully wetted soil as discussed in Section 2.1.4, the progression of heave with time at Profile A was calculated for ideal and poor grading conditions. The results are shown in Figure 8. The effect of surface grading is clearly shown in Figure 8. Whereas for ideal grading conditions only about 30 mm of heave will occur by the end of the design life, almost all of the potential maximum heave will occur by that time. This agrees with the experience of the authors in that in almost all cases where extreme structural distress has been observed, the grading around the structure has been poor.

Table 1 Summary of Soil Parameters Used in the VADOSE/W Models and Pier Design

Soil Type	Native Clay	Claystone
Saturated Hydraulic Conductivity, K_v (cm/sec)	1×10^{-4} (1)	6×10^{-6} (1)
K_h/K_v Ratio	1 (2)	10 (2)
Porosity	0.41 (3)	0.46 (3)
Percent Swell (%)	3.0 (4)	4.0 (4)
Consolidation-Swell Swelling Pressure (kPa)	335	480

Notes: (1) Calibrated values, (2) Assumed values, (3) Laboratory test data (4) Inundation pressure = 48 kPa

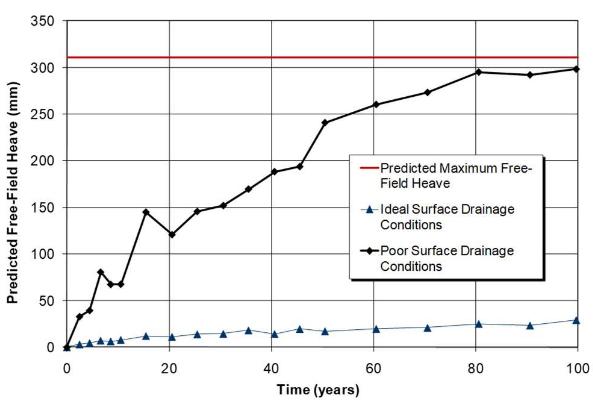


Figure 8 Predicted Free-Field Heave as a Function of Time for Ideal and Poor Surface Drainage Conditions

2.4 Effect of Grading on Deep Foundation Design

The required pier lengths for a deep foundation were computed for the ideal and poor grading conditions shown in Figure 6. The analysis used the design charts presented in Figure 4. Because the claystone along most of the pier length had uniform soil properties with depth, the pier-soil interaction was considered to correspond to Type A in Figure 4(a).

Design pier lengths for straight shaft piers and belled piers are compared in Figure 9. It is evident that the required pier length calculated by considering soil profiles having only partial wetting as a result of ideal surface drainage conditions is significantly shorter than the pier length calculated on the basis of fully wetted soil profiles resulting from poor drainage conditions. By controlling the surface drainage to maintain ideal conditions, the required design pier lengths can be reduced significantly.

3. VALIDATION OF THE FOUNDATION DESIGN METHOD BY NELSON ET AL (2015)

The design principles described above were validated using detailed soil properties and precise heave measurements on the Terminal Radar Approach Control building (TRACON) at Denver International Airport in Colorado, USA. The TRACON building is located on a site having highly expansive soils and had been undergoing heave movement for over 16 years. Construction of the TRACON building began in 1991 and continued into 1992. Heave due to the expansive soils was observed before construction was finished. Subsoils generally consist of 0.3 to 3.4 m of fill and native soils. The bedrock under the upper soils consists of weathered claystone underlain by interbedded or alternating layers of claystone and sandstone. Coal seams are interbedded in the bedrock. A typical soil profile and the primary subsurface units are shown in Figure 10. Monitoring of water migration and foundation movement was initiated in the year 2000 and continued through 2006. A detailed project history and information about the site are discussed in Chao (2007).

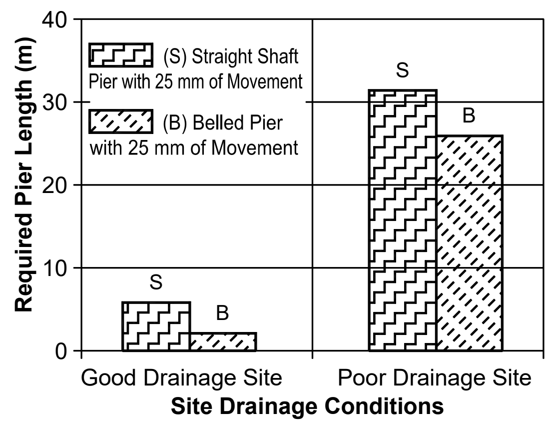


Figure 9 Required Pier Lengths for Both Good and Poor Drainage Conditions (Pier Diameter = 250 mm; Dead Load = 50 kN; Design Life = 100 yrs; Soil Profiles From Figure 6)

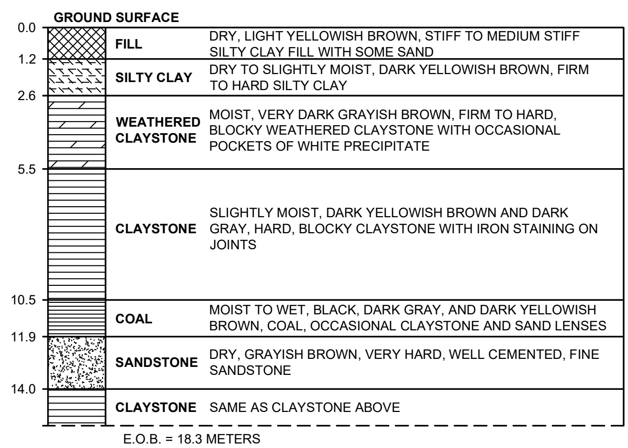


Figure 10 Typical Soil Profile and Primary Subsurface Units at the TRACON Facility

3.1 Water Content Profiles

Subsurface water content profiles were monitored by means of a downhole nuclear gauge. Access holes were cased with PVC tubes having a diameter of 50 mm and installed to depths from 3.0 to 24.4 meters below the ground surface. Access holes were installed at 12 locations. Monthly measurements were taken from June 2000 through 2004. Additional readings were taken in August 2006.

Water content profiles measured at two locations having very different surface conditions are shown in Figures 11 and 12. The profile shown in Figure 11 was measured in an area where irrigation had originally been applied, but was discontinued when the heave became intolerable. This was located near the corner of the building where the greatest amount of slab and pier heave had been observed. The profile shown in Figure 12 represents a contrasting condition. It is located underneath a concrete slab where climatic conditions other than temperature have little or no effect on the wetting of the subsoils.

It is seen in Figure 11 that water contents immediately above and below the coal seam were subject to seasonal fluctuation as the water in the coal seam fluctuated. The coal seam was seen to be a primary water source for the overlying claystone bedrock. Figure 12 shows that at the location under the pavement the water content in the claystone above the coal seam increased less than that shown in Figure 11. The coal seam shown in Figure 10 exists at a deeper depth, and is not believed to be continuous with the one shown in Figure 11.

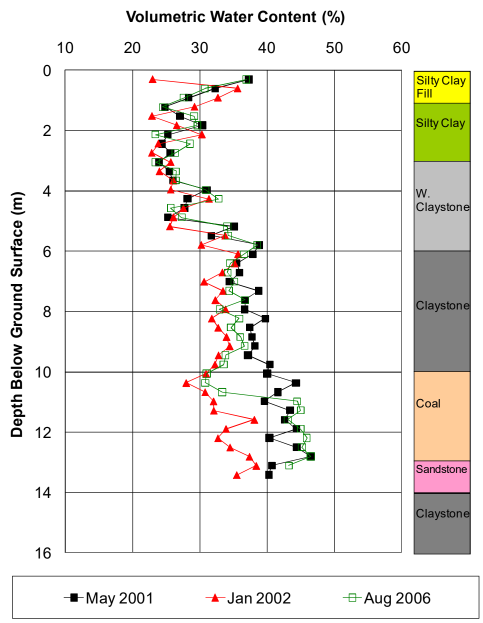


Figure 11 Volumetric Water Content Profiles in Bare Ground Area

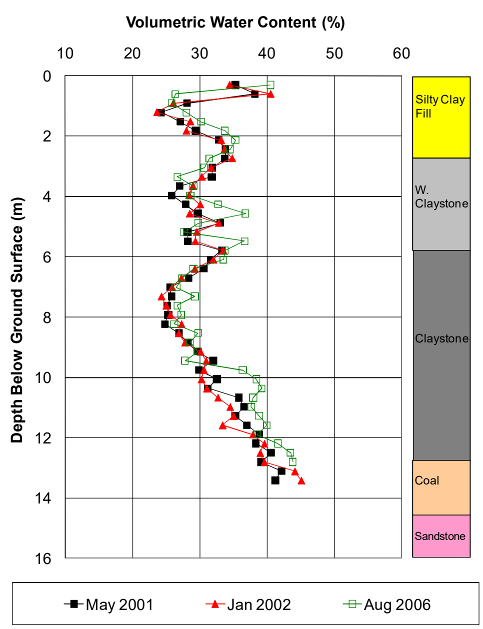


Figure 12 Volumetric Water Content Profiles under Pavement

3.2 Elevation Survey Data

Level surveys were performed at 50 floor and column locations throughout the building. An initial baseline survey was performed in September 2000. Surveys having an accuracy of ±0.5 mm were performed monthly for the first two years and quarterly, yearly, or bi-yearly after December 2002.

Three deep benchmarks were installed at the site to depths of 30 to 37 meters using procedures described in Chao et al. (2006). Relative elevations measured between all three benchmarks varied by less than 1.2 mm throughout the surveying period.

The amount of floor and pier heave that had occurred between the time of construction and the time of the initial baseline survey in September 2000 was calculated on the basis of the as-built elevations of the floor slab and/or the design. Figure 13 shows the maximum and minimum heave of the floor slab and the piers that occurred since the time of construction.

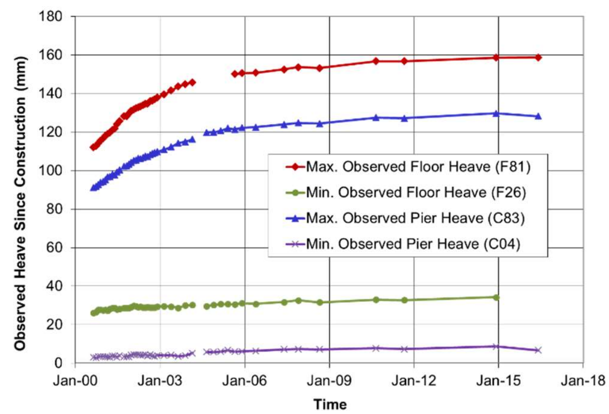


Figure 13 Observed Floor and Pier Heave since the Time of Construction

3.3 Migration of Soil Water with Time

Water migration in the vadose zone was analyzed for the two locations described above. The modeling was conducted for a time span corresponding to the 50 year design life of the foundation. The model input parameters and calibration are discussed in detail in Chao (2007). Figures 14 and 15 show the long-term water content profiles computed through the year 2040 for average precipitation conditions. Comparison of Figures 14 and 15 shows that with no irrigation at the ground surface, climate conditions significantly influence the top 6 m of the profile. This, then, represents the zone of seasonal fluctuation for the site. Several coal seams existed at the site, and these seams were a source of deep wetting. The deeper soils are significantly influenced by the coal seams.

3.4 Maximum Total Slab and Pier Heave Calculations

The future heave expected to occur from the time of sampling was calculated at each monitoring point location using the oedometer method that was presented above in this paper. The heave that had occurred from the time of construction to the time of sampling was added to the calculated future slab heave to determine the maximum total heave. Table 2 shows a summary of the properties of soil samples obtained from the borings used for the heave calculations.

The computed maximum total slab heave ranged from 150 mm near the western corner of the building to 264 mm at the easternmost corner. The maximum total pier heave was calculated to range from 81 mm at the northwest side of the building to 157 mm near the eastern corner.

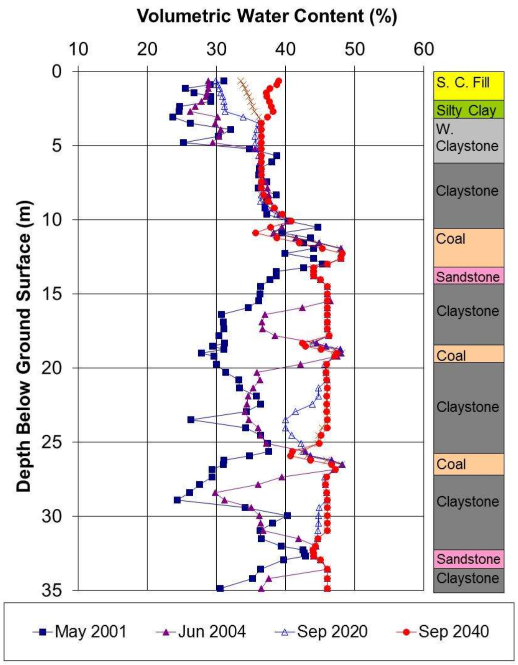


Figure 14 Calculated Long-Term Water Content Profiles for Bare Ground Profile

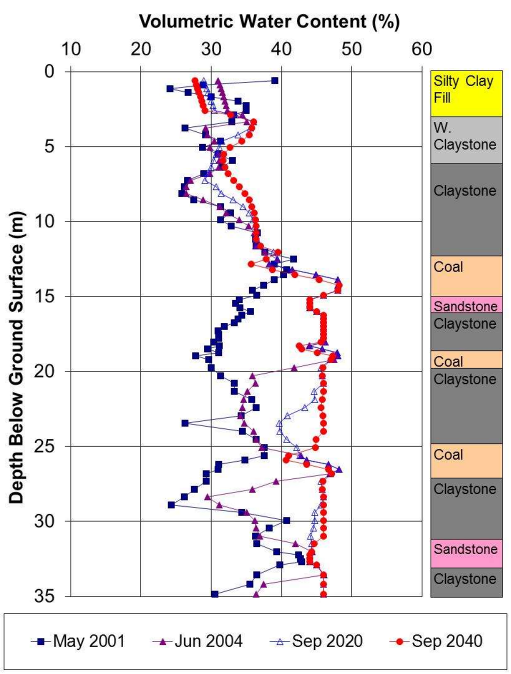


Figure 15 Calculated Long-Term Water Content Profiles Under Pavement Area

3.5 Computed Timewise Progression of Slab Heave

The progression of slab heave with time was computed by analyzing the migration of subsoil water in the manner presented previously, and then computing the heave corresponding to the changes of water content profiles. Heave was calculated for different points in time using the soil water profiles from the VADOSE/W modeling presented in Figure 14 taking into account the fact that not all soils would be fully wetted. The variation of slab heave with time was calculated for the soil profile at the location where the greatest amount of heave had been observed. This is labeled as Floor Monitoring Point F81. Heave was computed for scenarios of average precipitation both with and without irrigation.

Table 2 Summary of Soil and Bedrock Expansion Properties at the TRACON Facility

Soil Type	Natural Water Content (%)	Natural Dry Density (Mg/m^3)	Percent Swell (%)	Consolidation-Swell Swelling Pressure (kPa)
Silty Clay Fill	4.0 – 24.4	1.55 – 1.92	0.4 – 0.8 (1)	50 – 60
Silty Clay	19.1 – 21.1	1.68 – 1.75	-	-
W. Claystone	19.9 – 28.8	1.49 – 1.76	4.3 – 8.4 (1)	290 – 530
Claystone	7.4 – 31.5	1.30 – 2.08	3.0 – 10.2 (1)	290 – 1,400
Coal	20.3 – 40.8	0.88 – 1.57	-	-
Sandstone	11.5 – 19.9	1.68 – 1.91	-	-

Note: (1) Inundation pressure = 48 kPa

The slab heave computed using the procedure described above is presented in Figure 16. Without irrigation, the maximum slab heave was calculated to reach a value of about 175 mm at the end of the design life of the building in 2040. This value is only about 66% of the calculated maximum total slab heave. Figure 16 also shows that with irrigation at the ground surface the heave is expected to approach the calculated maximum total slab heave during the design life.

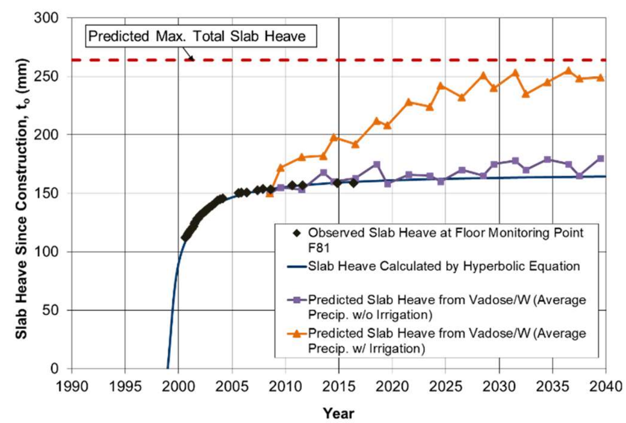


Figure 16 Observed and Calculated Slab Heave as a Function of Time – Floor Monitoring Point F81

3.6 Computed Timewise Progression of Pier Heave

The progression of pier heave with time at the location of Pier Monitoring Point C98, which is located next to Floor Monitoring Point F81, was calculated in this study. The pier was constructed to be 8.5 meters in length and 0.6 meters in diameter at this location. The water migration pattern for the TRACON site is somewhat complicated. As shown in Figures 14 and 15, the coal and sandstone layers were the major water sources to the subsoils. The water migrates not only downward but also upward from the coal and sandstone layers. This kind of complex groundwater migration pattern precludes the usage of the design charts for the pier heave calculations. Therefore, the APEX program was used for the calculations.

The results of the slab heave with time for the scenario of average precipitation without irrigation shown in Figure 16 were used in the pier heave calculations. The results of the calculated pier heave with time are shown in Figure 17. The maximum pier heave was calculated to reach a value of about 80 mm at the end of the design life of the building in 2040. This value is about 88% of the calculated maximum total pier heave.

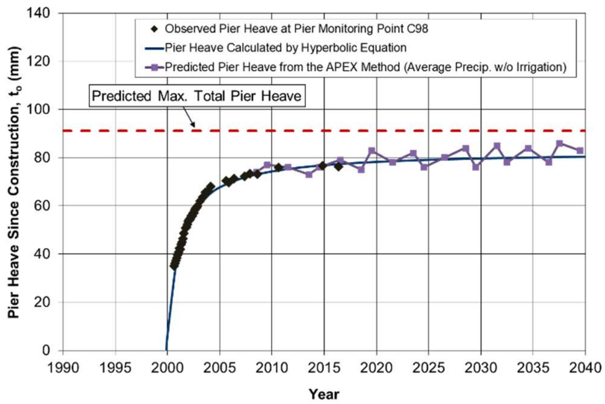


Figure 17 Observed and Calculated Pier Heave as a Function of Time – Pier Monitoring Point C98

3.7 Projection of Measured Slab and Pier Heave

In order to compare the time-wise progression of the calculated slab and pier heave with the measured slab and pier heave, the survey data were projected forward in time by fitting them to a hyperbolic function of the form shown in Equation (3).

$$\rho = \frac{t}{a + bt} \tag{3}$$

where: ρ = slab or pier heave since the time of construction; a and b = curve parameters; and t = the time since movement began.

One problem in analyzing the data in this manner was that the time at which heave at the ground surface actually began, t_o, was not evident just by observation. To determine an appropriate value for t_o, several plots of the measured data were made assuming different dates for t_o. The correct date for t_o was taken as the one corresponding to the particular curve for which the plotted data exhibited the best value of the correlation coefficient, r^2. The dates when all of the floor monitoring points began to heave using that technique ranged from January 1st, 1991 to December 1st, 1993. These dates are consistent with observations of the time required for groundwater to migrate across the site.

Using the same technique as described for the slab heave calculations, the dates when the piers began to heave were determined to range from January 1st, 1994 to December 1st, 1997. In comparison with the values of t_o for the slab, inception of pier heave lagged the slab heave by about 4 to 5 years. This is to be expected because as the zone of wetting increases, the length of the pier that is within the active zone increases, and the length of pier in the anchorage zone decreases (Nelson et al., 2001).

The extended survey data for the slab and pier heave based on Equation (3) are shown as the solid line on Figures 16 and 17, respectively. The maximum projected slab heave shown in Figure 16 was equal to 164 mm at the end of design life. This is only 62% of the computed maximum total slab heave. The projected pier heave at the end of the design life shown in Figure 17 was equal to about 80 mm, which is about 88% of the maximum total pier heave. It is of particular interest to note that in Figures 16 and 17 the calculated heave agrees very well with the projection of the measured heave using Equation (3).

3.8 Accuracy of Slab and Pier Heave Prediction

Examination of Figure 16 shows that whereas the slab heave predicted using VADOSE/W modeling is greater than that predicted by the hyperbolic fit after the year 2025, the general trend of the predicted heave using both methods are similar. It is believed that heave prediction based on water migration is more realistic in that it considers actual soil profiles, soil properties, and climate conditions. Also, a relatively long period of actual heave monitoring is not needed for that method.

Comparing the calculated heave with the projected measured heave shown in Figures 16 and 17 indicates that with careful analysis heave can be computed within an accuracy of about 30%. Considering the inherent variability of geotechnical engineering parameters, this degree of accuracy is quite good.

4. DISCUSSION AND CONCLUSIONS

Current design procedures for foundations on expansive soils generally are based on the calculated heave that can ultimately occur at a site. At sites with highly expansive soils, it is often impractical to design a foundation system for the maximum heave. It is more practical and cost effective to consider the migration of water within the foundation soils and the heave that such wetting will produce over time. The design method set forth in this paper provides a practical and economical approach to the design of foundations on expansive soils. It demonstrates the manner in which the migration of the subsurface water should be taken into account. The required pier length may be reduced significantly if it can be shown that the full depth of potential heave is not expected to be fully wetted.

Unless site specific analyses can be performed to accurately determine the rate and pattern of subsoil wetting, a prudent designer should assume that the entire depth of potential heave can become wetted during the design life of the structure. This may often be the case if the depth of potential heave is not large.

Grading and irrigation practices are important factors. Proper practices can be recommended to owners of structures, but it must always be recognized that they are likely not to maintain the property in a manner that is consistent with the engineering design criteria. Thus, the design engineer must consider the risk that maybe imposed by poor maintenance practices.

The progression of slab and pier heave for the TRACON building was analyzed by curve fitting of measured data to a hyperbolic equation. It was also analyzed by computer modeling of the subsoil water migration and calculating heave as the water content changed. It was shown that by the year 2040, which represents the design life of the TRACON building, the slabs and piers are expected to heave by about 62 to 88% of the predicted maximum total heave.

The predicted maximum total slab and pier heave calculated using the Nelson et al. (2015) method represents the maximum potential heave that could occur if the soils/bedrock are fully wetted to the entire depth of potential heave. The extrapolation of the measured survey data using the hyperbolic fit assumes that the wetting in the future will encounter a soil profile and degree of wetting similar to that which has been wetted to the present time. Actual heave rates will be influenced by non-uniform heave resulting from the wetting of various soil strata, fluctuations of climate conditions at the ground surface, redistribution of soil water within soil strata, and possibly other factors. It is expected that they will not precisely follow a smooth hyperbolic function over time. Therefore, the actual ultimate heave is expected to be bounded by the maximum calculated heave and the heave determined by using the hyperbolic fit.

The extensive program of monitoring and analysis conducted at the TRACON building at Denver International Airport provided a unique case history on which to validate the design method presented in this paper. It was shown that the heave over the period of the analyses could be predicted with good accuracy. Achievement of that degree of accuracy required careful soil sampling and testing, and rigorous analyses. It was shown, that with careful drilling, sampling, and soil testing along with rigorous analyses, the design principles presented herein will provide a reasonable and safe foundation design.

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